SECTION 3
INTEGRITY INVESTIGATION

General

As introduced in Section 1, the objectives of a feasibility level integrity investigation of an existing impoundment for the addition of hydroelectric facilities are (1) to determine the structural conditions and hydraulic performance characteristics of the dam, reservoir, and appurtenant works; (2) to assess their capability of being utilized safely for small hydroelectric power generation; (3) to determine the nature and to estimate the cost of any remedial measures necessary for such safe utilization; and (4) to estimate their longevity and future maintenance needs while serving that purpose.

These objectives are most readily achieved by conducting the investigation in from one to three stages. In Stage 1 the dam, reservoir, and appurtenant works and all existing records pertaining to them are examined, reviewed, and evaluated. In Stage 2 supplemental data and analyses are acquired and evaluated and conclusions are made concerning the integrity of the impoundment and any need for remedial repairs or alterations. In Stage 3 the alteration and repair schemes for any necessary rehabilitation are conceived and their costs estimated. The remaining useful life of the facilities and the associated annual maintenance costs are also determined. The need and scope of Stages 2 and 3 are determined by the Stage 1 findings. Should addition of power facilities prove feasible, additional detailed investigations and analyses are carried out at the design level, but discussion of these is not within the scope of this volume.

Stage 1 - Review of Existing Data and Site Reconnaissance

General. The purpose of Stage 1 is to make an initial evaluation of the integrity of the existing facilities by maximum utilization of all available records and by detailed on-site examinations. One of the objectives of Stage 1 is to determine whether or not Stage 2 is needed for a final evaluation and to establish the scope of that stage. Rarely, it may be possible to proceed directly to Stage 3. Occasionally the Stage 1 findings may dictate that the investigation be terminated.

Review Existing Data. The investigation logically and purposely commences with assembling, organizing and reviewing all information that is already available concerning the facilities. Among the important questions which should be addressed are the following:

- How were the facilities designed?
- What were the loading assumptions?
- What engineering properties were assigned to the construction materials and the foundation?
- Were they based on laboratory and field tests?
- What were the test procedures?
- Were they reliable and representative of actual service conditions?
- What criteria were imposed for stress and stability analyses and what were the actual results?
- How were the flood-producing characteristics of the drainage basin evaluated?
- What runoff records were then available?
- How was the inflow design flood developed?
- How do the peak flow and volume of the hydrograph compare with the envelope values for other hydrologically similar basins in the region?
- What have been the record flows at the facility since completion?
- What kinds of construction procedures and methods were used?
- What were the corresponding technical provisions of the contract specifications?
- What were the specified construction materials properties and characteristics?
- How was quality control maintained and measured?
- What engineering inspections were made during construction?
- What were the actual conditions encountered in exposing the foundations?
- What design changes were made to conform to those conditions?
- What has been the performance record to date, as revealed by instrumental observations and reports of past inspections?
- Have any repairs or alterations been necessary? Why?
- How were they made?
- Has the dam ever been raised? How?

Answers to the foregoing questions and others can be obtained in varying degrees from records, if they were made and can be found. Depending upon their quality and completeness, they can be of great value in initially evaluating the structural and hydraulic suitability of the facilities. In any event, advance study of whatever records are available (such as previous inspection reports) will provide selective guidance to the inspecting personnel during their on-site examinations. Those records may also provide basic data such as material test results and foundation exploration information for use in the engineering analyses to be made in Stages 2 and 3. The need for and nature of additional basic data for Stage 3 will also be determined by the kind and quality of the data found in the records.

The records on existing dams vary considerably in completeness, quality, and usefulness. Their existence and character will vary with the age of the facilities, the type of ownership, and the project engineer, if there was one. In many cases, records (especially of design and construction) may be totally nonexistent, fragmentary, or inaccurate. It is important, however, that a diligent search be made for all records, because the information therein may be vital and unavailable from any other source, e.g., treatment of unusual or difficult foundations.

The search for records should include the files of the owner, of his engineer (in-house or retained), and of supporting specialists such as geotechnical engineering firms and consultants. Rarely records may be available from construction contractors. State agencies administering effective dam safety programs will have accumulated past records and they maintain current records. Their files may ease the investigator's search for records and be highly informative. Useful information may be reported in volumes of periodicals such as *Engineering News Record,* especially for older dams.

Answers to questions like those mentioned previously and other disclosures essential to the investigation will be found in engineering design and construction records often descriptively and conventionally titled with regard to their original purpose and use. Of course, the quality and accuracy of the engineering reported by the record must be examined and used by the investigator with discrimination and not uniformly accepted at face value. (For example, the drawings may not show actual, as-built conditions.) A reasonably comprehensive list of records and reports categorically grouped is presented herein. Such complete records will be rare for the dams being investigated.

---

1. Design records -

Contract plans and specifications

Geologic report

Site and materials exploration report

Design report or design bases (methods of analyses, analyses assumptions, assigned materials and foundation properties, stress and stability summaries, spillway design flood, flood routing summary, etc.)

Materials testing and appraisal report

Site seismicity report

Designers' operating criteria

Stress model reports

Hydraulic model reports

Technical record of design

2. Construction records -

Photographs - especially of foundation surfaces and preparation

Daily inspector's reports and construction progress reports - especially for descriptions of foundation and construction materials quality, unusual treatment and preparation, contractor's compliance with technical provisions of the specifications, etc.

Record of foundation drilling and grouting, and contraction joint grouting

Quantified materials quality control record of embankment and concrete engineering properties

Weekly, monthly or other periodical or special interim reports

Final construction report

Final geology report

Final grouting report

Instrumentation installation report and record of measurements during construction to establish baseline data

3. Reservoir operation records -

Chronological reservoir stages - especially for unusual stages, noteworthy spillway and outlet discharges, taxed spillway capacity, etc.

Standard operating procedures - especially for unusual, difficult, or uncertain functioning of gates, valves, controls, etc.

4. Performance record -

Hydraulic performance records of the separate spillway and outlet components at different stages and discharges.

Instrumentation design, layout and records, observation program, schedule, chronological plots, etc.

5. Maintenance record -

Reports of previous inspections, including photos of both normal and unusual conditions.

Recent evaluation reports of structural and hydraulic conditions and recommendations for remedial work or operational requirements and restrictions.

6. Records of significant past repairs, raises or alterations -

Correspondence files over the life of the facilities commencing with the design period may contain clues concerning the integrity of those facilities.

Basic Data Studies. Besides studying records relating to the dam in question, available data relating to the area and site (which may or may not have been available or used in the original or subsequent work on the dam) should also be reviewed. It must be determined how (if at all) this current knowledge modifies the conditions that must be considered in the dam's operation as a hydroelectric facility. This study of available hydrologic, meteorologic, geologic and seismic data should be performed prior to the dam inspection to form a frame of reference for the inspection.

Conduct Site Inspections. Inspections of existing impoundments are most intelligently made when the inspector is armed with the knowledge obtained from the record; guided by an understanding and familiarity with the way structures behave under various loads and water flows; informed on the way materials and natural formations react to their environment; and acquainted with actual modes of accidents and failures and their underlying causes.

Experience has revealed general classes of concerns meriting integrity investigation, especially of older impoundments in the size ranges appropriate for the addition of small hydropower. A whole host of specific conditions creating the concerns have been identified within these classes. For a comprehensive investigation, the principles expressed by these general classes must not become obscured while concentrating on specific details. The following general classes of concern prevail at all types and sizes of dams:

1. Ability to handle expected inflow floods
2. Stability of the dam and other water barrier structures under all anticipated forces and modes of operation.
3. Stress ranges in the dam and other structures critical to impoundment and operation of the reservoir.
4. Hydraulic capability of the outlet works
5. Load supporting capability of foundations
6. Control of seepage, leakage, and erosion in dam, foundation, and the confining boundaries of the reservoir
7. Deterioration of materials and foundation
8. Reliable service and operation of spillway and outlet control devices.

Failure modes and causes have been reported and discussed extensively in engineering literature (ICOLD, 1973; ASCE, 1975; Biswas, 1971). They are discussed in a general manner in Section 2.

The inspecting party should be comprised of a group of qualified, professional personnel, educated and experienced in dam design, construction and inspection. The number and discipline of the members are determined by the type and complexity of the structure and the reservoir environs. A civil engineer and engineering geologist would be a minimum-sized party. A mechanical engineer would be included dependent upon the type and complexity of the installed mechanical equipment. The individual responsible for making the final integrity evaluation should be a member of the party whenever possible. In many instances he would be the civil engineer member. A civil engineering specialist (a soils engineer, a concrete specialist, a structural engineer, a hydraulic design engineer or others) may be needed. The owner's project operation and maintenance personnel familiar with the facilities should be present to assist the party and supply information from their experience and knowledge. If the owner has an engineering staff, an individual from the staff should be present.

Checklists are often helpful to the inspection party for guidance and as "memory joggers". In principle, a checklist tabulates separately identifiable components of the dam, appurtenant works and other project features that merit observation for structural and hydraulic behavior, durability of materials, stress, strain, stability, seepage, leakage, drainage, erosion, operational capability and reliability, cavitation, temperature response, performance, instrumentation capability and serviceability, and maintenance. Checklists also are reminders for obtaining general information associated with the inspection itself, such as participants, project access, and communications. A checklist can be prepared in advance and tailored to each specific impoundment, using the information obtained from the review of the record, while keeping in mind the general classes of concern discussed earlier in this section. Reference to a universal checklist (Exhibit I of this volume) will help make the specific checklists complete.

While a checklist may be a useful tool in the hands of a knowledgeable person, it may mislead, confuse, or inhibit unqualified or inexperienced personnel by limiting the scope and detail of the inspection. Checklists are of little value unless the party members know what to look for, how to interpret what is visible, and how to make an evaluation based on indirect as well as direct evidence. Interpretation and evaluation of the observations are done by the application of engineering principles and judgment. Completion of a checklist should not be regarded as a selfsufficient measure of evaluation.

Evaluation of Data and Formulation of Conclusions.

General. The preliminary evaluation of the integrity of the impoundment is made by collectively considering all pertinent information revealed by the record, all conditions observed at the site, and the results of those engineering analyses that can be made by the investigator with the existing record data and by his checks of any recorded analyses. Engineering judgment by individuals experienced in dam design and construction is essential in the process.

If the Stage 1 preliminary evaluation is favorable in all respects, the feasibility study described in Volume I may proceed without Stages 2 and 3 of the integrity investigation. If the preliminary evaluation is favorable and can identify positively the specific rehabilitation needs, Stage 3 may follow directly. If the preliminary evaluation is uncertain but promising, Stage 2 should follow. If Stage 1 or Stage 2 evaluations are clearly unfavorable, the investigation should be terminated at the completion of those stages after consultation with the responsible project managers.

The engineering analyses portions of the evaluation process are discussed in this section on Stage 1, even though all the needed analyses may not be possible during Stage 1. Presentation here, then, not only includes analyses to be made during both Stage 1 and Stage 2 but also serves to identify additional data required for Stage 2 (See discussion below on Stage 2).

Standards and Engineering Criteria. In order to decide whether a dam and its appurtenant works can, in fact, safely store and control flows of water, an investigator must apply some measure of adequacy. Because there are many associated considerations, both direct and indirect, these decisions can seldom be made based solely on the application of rigid "standards" and engineering criteria. A "standard" as used here is considered to be a definite rule established by authority (usually governmental regulation), while a criterion is considered a test of quality by the application of engineering principles. The statutes, codes, and regulations of governmental agencies having various kinds of jurisdiction over such matters as water rights, public safety, environmental protection, or occupational health and safety may be controlling or contradictory. How, then, do they apply in the case of this type of study? What is the pertinence of the state of the art as practiced when the dam was designed and constructed compared to that existing today, especially if the facility has ably performed over the years? What are the relative hazards (source of potential danger created by the existence of the dam and reservoir) when compared to the degree of risk (the probability of failure and the chance of loss of life and property) that exists at the time of the study? What is the influence of public opinion and the public's demonstrated unwillingness to accept involuntary risks (Starr, 1969) and how are they relevant to the study's conclusions? How does the owner knowingly view his liability? What is the liability of the evaluator? What are today's commonly accepted practices for designing, constructing, operating, and maintaining dams, and how are they influenced by conflicting schools of thought among different groups of engineers? Yesterday's standards may prevail when evaluating liability for a failure of an old structure but new standards will prevail for an altered structure. All these questions are legitimate and must be considered in evaluations such as are covered by this volume. The blind application of standards or criteria is not adequate.

The use of standards and criteria as measures of adequacy can be dangerous, biased, or restrictive when numbers and specific values are generated by an engineering analysis and then compared as a pass or fail test. Of far greater importance than the numbers that are generated by the analysis is the evaluator's understanding of the degree of accuracy of the values and the assumptions going into the analysis, the limitations of the analysis, and the true representation of actual conditions. The interpretation and application of the numbers from the analysis must be tempered with common sense, understanding, experience, and judgment.

Instead of basing his evaluation on just barely meeting some imposed minimum standard, the investigator should make his evaluation based on demonstrated sound engineering practices generally endorsed by the collective dam engineering profession, coupled with his own experience and convictions. Competent, conscientious investigators will usually match or exceed the so-called standards without being unduly conservative.

Methods of Analyses. Many analytical techniques - mathematical, graphical, and physical (models) - have been developed for investigating and predicting the behavior and response of dams, other hydraulic structures, and their foundations in different physical environments and service conditions under all kinds of loading. These techniques are used to help find dependable answers to the general classes of concern introduced above.

These techniques are available in prolific detail with examples from many sources - university textbooks for fundamentals; professional engineering society publications such as United States Committee on Large Dams (USCOLD), International Congress on Large Dams (ICOLD), and American Society of Civil Engineers (ASCE) for practical specific applications; design manuals, monographs, handbooks, and design standards of federal and state agencies engaged in water resource development for methodical, production-basis use — for example, publications of the U.S. Army Corps of Engineers (COE), especially the Hydrologic Engineering Center; technical publications of product manufacturers and construction materials associations such as the Portland Cement Association (PCA), the American Concrete Institute (ACI), the American Institute of Steel Construction (AISC), and the American Concrete Pipe Association (ACPA), the Stress Steel Corporation, ARMCO Drainage and Metal Products, the American Asphalt Institute, etc. for detailed analytical methods of hydraulic structure components where their products are used (ACPA, 1957; ACPA, 1959; ARMCO, 1955). Several publications (Golze, 1977; Justin, 1945) are outstanding. Some (USBR, 1974/2; NRC, 1939) are also especially suited as well to the size class of dams having potential for small hydropower. Private engineering firms specializing in hydraulic project planning and design have developed manual-like compilations for their in-house use.

Analyses most frequently and conventionally made for reservoirs, dams, and appurtenant structures in size ranges which may be candidates for small hydro investigations are:

1. Inflow design flood hydrograph (COE, v.3 April 1975; and v.5 March 1975).
2. Reservoir flood routing (COE, v.4 October 1973, and v.7 February 1976).
3. Spillway discharge rating curve (USBR, 1974/2, Sections 195-200, 211-214.)
4. Open channel water surface profile (USBR, 1974/2, Sections 203-204).
5. Tailwater elevation-discharge curve (USBR, 1974/2, Section B-8, B-9).

6. Outlet discharge rating curve (USBR, 1974/2, Sections 222, 232-236, B-3).

7. Hydraulic jump characteristics of stilling basin (USBR, 1974/2, Sections 205-210).

8. Water surface profile in the trough of side-channel spillways (USBR, 1974/2, Section 202).

9. Trajectory of overflowing nappe (USBR, 1974/2, Section 211) or free falling jet.

10. Plunge pool scour depth (USBR, 1974/2, Section 210).

11. Conduit (penstock) pressure surge.

12. Buoyancy resistance, stability, stresses for free-standing dry-type intake tower.

13. Active, passive, at rest earth pressure.

14. Retaining wall, spillway gate pier, spillway control structure stability, stresses, deflections.

15. Stresses, deflections, reactions in spillway and outlet control devices (radial gates, flashboards, etc.) and in anchorages.

16. Stresses in conduits (Beggs, 1968; USBR, 1965).

17. Stability of embankment and foundations (ASCE, 1969; Janbu in Hirschfeld, 1973, pp. 47-86).

18. Stability of natural formation confining the reservoir.

19. Stability of hillside adjacent to abutments.

20. Seepage flow nets for pore pressures, hydraulic gradients, and escape gradients in embankments and foundations (Cedergren in Hirschfeld, 1973, pp. 21-45).

21. Consolidation, subsidence, compression, and expansion of foundations.

22. Stability and stresses in concrete gravity sections (dams, locks) (Golze, 1977, pp. 385-393, 437-445, 583-587; Justin, 1945, pp. 247-423; USBR, 1976, Chapters 2 and 3).

23. Stresses in arch dams (Golze, 1977, pp. 385-437; Justin, 1945, pp. 425-553; USBR, 1977, Chapters 3 and 4), arch barrels (Justin, 1950, pp. 584-587), facing slabs (Justin, 1945, pp. 558-599).

The determination of the particular analyses that must be made will of course depend upon the type of dam, its age, observable conditions, performance history, watershed, stream, and reservoir characteristics, geologic setting, etc. For example, reservoir flood routings serve no useful purpose if the reservoir capacity is small and the drainage basin is large. Dynamic stability analyses are unnecessary in regions of low seismicity. The cost and time required will be reduced if simpler methods of analyses can be used. Refined procedures and precise results are not always needed in order to make a decision. The more experienced the analyzer, the fewer the analyses that may be needed. Analyzers who are generally knowledgeable but inexperienced need more data and studies to make evaluations. Evaluations by unknowledgeable persons will not only be inaccurate but can lead to false conclusions as well as dangerous expectations.

The detailed developments, explanations, instructions, applications, and examples of these analyses, some of which can be made by several different accepted methods, will be found in the selected references.

Of these many analyses the ones usually considered most critical for integrity investigations are: (1) those concerning adequate spillway capacity or, more generally stated, the ability to safely handle expected inflow floods; and (2) those concerning the stability of the dam and foundation for safely impounding the water in the reservoir. Because of their importance, these topics are discussed in more detail below.

Ability to Safely Handle Expected Inflow Floods. The ability of the impoundment to safely handle expected inflow floods first requires preparation of an inflow flood hydrograph or peak inflow value on some acceptable frequency or probability-of-occurrence basis. If detention storage capacity is operationally reserved for that purpose, the inflow flood is routed to determine the residual freeboard protecting non-overpour structures. The hydrologic techniques for flood estimating and routing are discussed in Volume III. Criteria for the flood magnitude and residual freeboard are discussed in a later segment of this section, "Suggested Engineering Criteria."

Certain investigations of spillway capability can be made by analytical methods. The spillway rating curve is calculated for use in the flood routing study. Usually the capacity will be established by the control structure but any other components that might become capacity-controlling, usually at higher discharges, must not be overlooked. For example, at a double side-channel spillway, the hydraulic control may shift to locations in the side-channel trough or to the juncture of the trough and the discharge channel. The control may shift from free-surface flow to orifice flow to pressure flow at shaft and drop-inlet spillways. The water surface profile in an open channel can be calculated to investigate wall overtopping. Cross-channel wave patterns created by channel convergence or curvature can be determined (at least qualitatively) for the same reason. The hydraulic jump characteristics or nappe and jet trajectories at the terminal structure can be calculated to investigate energy dissipating capability. The tailwater rating curve can be calculated from a known downstream hydraulic control to investigate the effect of the tailwater elevation on flows and on the terminal structure.

An impoundment may not have a spillway and it must then be investigated for ability to temporarily store the inflow volume and dependably draw off that volume through available release facilities before succeeding floods occur. In such cases the investigation of the capacity, structural integrity, and operational reliability of all components of the release facilities used for that purpose becomes of great importance.

Stability of a Dam and Foundation. Certain investigations for the stability of a dam and its foundation can be made by analytical methods, dependent upon the dam type.

Embankment-Type Dams. Various methods of slope and foundation stability analyses are available. The more common ones are two-dimensional and are based on limiting equilibrium. These analyses are known by a variety of titles, including slip circle, Swedish circle, Fellenius method, method of slices, sliding block, etc. There are differences in assumptions and force resolutions in the different methods. When forces representing earthquake effects are included, the analysis is often termed pseudostatic. The analysis is made by assuming some form and location of failure surface such as a circular arc, compound curved surface, or a series of connected plane surfaces. The configuration and positioning of the surface depend upon the kind of embankment dam, the internal zoning, and the foundation geologic structure. For example, connected plane surfaces are often used for an inclined or sloping core rockfill dam. The trial failure surfaces are positioned judgmentally to pass through weaker or more highly stressed regions. For example, a plane surface may be positioned in shallow weak clay or in shale layers in the foundation; or a circular surface may be positioned in a confined fluvial foundation susceptible to high pore pressure. The most critical surface is defined as the one having the least computed factor of safety which is considered to be the ratio of forces or moments resisting the movement of the mass above the surface being considered to the forces or moments tending to cause movement. Both embankment slopes are analyzed for the specific service conditions expected. The most critical case for the downstream slope is usually full reservoir with steady seepage; for the upstream slope it is usually either rapid drawdown or reservoir partially full with seismic loading. Seismic cases for these methods of analyses assume horizontal loads determined from constant horizontal seismic coefficients whose values are arbitrarily selected on the basis of ground motions anticipated at the site. The engineering properties and strength values used in these analyses must be selected to duplicate as closely as possible the actual field conditions expected. For example, if drainage during the application of forces is not possible, shear strengths should be based on quick or consolidatedundrained laboratory tests. These analytical methods can also be used to examine reservoir and abutment hillside slide potentials. More realistic but extensive and costly dynamic analysis methods are available for investigating the effects of earthquakes on stability. These methods are based on limiting strains and permanent displacements rather than factors of safety. Only in very special situations would such analyses be employed in small hydro investigations. Instead, simplified procedures (Makdisi, 1978) for estimating the earthquake-induced deformations are available if needed.

Allied analyses are used during stability studies to determine seepage patterns and amounts, pore pressures, uplift forces, hydraulic gradients, and escape gradients in the embankment zones and the foundation by the application of the principles of flow through porous media and the graphical or mathematical modeling of flow nets (Cedergren, 1977).

Concrete and Masonry Dams. The stability of gravity dams and the buttresses of buttress-type dams can be numerically evaluated for resistance to sliding and overstressing from water, weight, uplift, earth and silt, temperature, seismic, and ice loads. The resistance values are calculated on critical surfaces in the dam, on the foundation, and below the foundation level. The resistance to overturning can also be calculated, but any indicated instability will most likely be manifested by local crushing of the concrete or the foundation due to overstressing, rather than a physical toppling of the intact mass. The principles and procedures of these analytical methods are also applicable to lock walls, spillway control structures, and retaining walls.

The stresses in the arches and slabs of buttress-type dams and arch dams can be numerically evaluated for the same kinds of loads as for gravity dams.

Single-arch dams may be further characterized as being constant radius, constant angle, variable radius, or double curvature. The arch rings may be cylindrical and of uniform thickness or of irregular form and of variable thickness.

Depending upon the height, geometry, complexity, and importance of an arch dam, the stresses can be approximately determined by the cylinder theory (NRC, 1939; Justin, 1945, pp. 425-553) or by the application of the theory of elasticity using graphical and mathematical summation methods (Justin, 1945, pp. 425-553). Various assumptions and considerations can be included or omitted that will affect the complexity of the calculations and the relative validity of the resulting stresses. Two examples are deformations due to shear and the effect of Poisson's ratio. The arch rings may be considered fixed or hinged at the abutments. The abutments may be considered rigid or elastic. Contraction joints may be considered grouted or ungrouted.

The more realistic and exacting methods of trial-load analysis and two- and three-dimensional finite element analyses are available. Stress patterns in the abutment and foundation mass of concrete dams can be determined by the finite element methods. Such analyses will not usually be necessary in small hydro investigations; however, where special and critical situations exist, these types of analysis may be justified.

Suggested Engineering Criteria. "There was unanimous agreement that it would be unwise to publish recommended design criteria as standards to be adopted and used universally... Consequently, it could be extremely dangerous to publish design criteria and thereby imply that by following these criteria an engineering organization could assure that a safe structure will result" (ASCE, 1967). Although these words were written about the design of "large" dams they are equally applicable to the investigation of the integrity of smaller dams. This referenced joint ASCE-USCOLD committee report summarizes the practices for dam design and construction of major engineering organizations in the United States and provides excellent criteria statements for use here.

Criteria are sometimes stated on the basis of dam size and the related hazards and risks. There is no universally accepted definition of a "large" dam. Hazard is a function of dam size and physical condition. Risk is a function of potential project damage, monetary loss, and of population location and density. Definitions that have been suggested indicate that dams appropriate for small hydro are of small and intermediate size.

The U.S. Army Corps of Engineers has recommended (COE, 1977) expected inflow flood magnitudes for use in the National Program of Inspection of Dams. Those recommendations which are appropriate for guidance here are excerpted from that reference and presented as Tables 3-1, 3-2, and 3-3. There are some differences in terminology for floods, hazards, and risks but the interpretations are obvious.

TABLE 3-1
RECOMMENDED SPILLWAY DESIGN FLOODS[a]

Hazard[b]	Size[c]	Spillway Design Flood (SDF)[d]
Low	Small	50 to 100-yr freq
	Intermediate	100-yr to 1/2 PMF
Significant	Small	100-yr to 1/2 PMF
	Intermediate	1/2 PMF to PMF
High	Small	1/2 PMF to PMF
	Intermediate	PMF

[a] Source: COE, 1977.

[b] See Table 3-2 for definitions

[c] See Table 3-3 for definitions

[d] The recommended design floods in this column represent the magnitude of the spillway design flood (SDF), which is intended to represent the largest flood that need be considered in the evaluation of a given project, regardless of whether a spillway is provided; i.e., a given project should be capable of safely passing or storing the appropriate SDF. Where a range of SDF is indicated, the magnitude that most closely relates to the involved risk should be selected.

100-yr = 100-Year Exceedence Interval. The flood magnitude expected to be exceeded on the average of once in 100 years. It may also be expressed as an exceedence frequency with a one-percent chance of being exceeded in any given year.

PMF = Probable Maximum Flood. The flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. The PMF is derived from probable maximum precipitation (PMP), which information is generally available from the National Weather Service, NOAA. Most Federal agencies apply reduction factors to the PMP when appropriate. Reductions may be applied because rainfall isohyetals are unlikely to conform to the exact shape of the drainage basin and/or the storm is not likely to center exactly over the drainage basin. In some cases local topography will cause changes from the generalized PMP values, therefore, it may be advisable to contact Federal construction agencies to obtain the prevailing practice in specific areas.

TABLE 3-2
HAZARD POTENTIAL CLASSIFICATION[a]

Hazard Category	Loss of Life (Extent of Development)	Economic Loss (Extent of Development)
Low	None expected (No permanent structures for human habitation)	Minimal (Undeveloped to occasional structures or agriculture)
Significant	Few (No urban developments and no more than a small number of inhabitable structures)	Appreciable (Notable agriculture, industry or structures)
High	More than few	Excessive (Extensive community, industry or agriculture)

[a] Source: COE, 1977.

TABLE 3-3
SIZE CLASSIFICATION[a]

Category	Storage (Ac-Ft)	Impoundment Height (Ft)
Small	50 to 1,000	to 40
Intermediate	1,000 to 50,000	40 to 100

[a] Source: COE, 1977.

The U.S. Bureau of Reclamation has published (USBR, 1974/1) design criteria for concrete arch and gravity dams. Those criteria are appropriate for use here. The subject matter is organized and presented in brief, systematic fashion by first discussing each basic consideration and then making the criterion statement. Loads and load combinations, safety factors and their application limitations, assumptions and uncertainties of analyses and materials properties, limiting stresses, and minimum stability factors are all presented.

Two excellent publications (John Lowe III, in ASCE, 1969, pp. 1-35; Nilwer Janbu, in Hirschfeld, 1973, pp. 47-86) comprehensively discuss the state of the art and the mechanical principles for embankment-type dam stability analyses by limiting equilibrium methods. Although minimum factors of safety criteria are not presented, an appreciation and understanding of the advantages and limitations of the methods of analyses can be obtained from which the investigator for small hydro can better understand why it is no simple matter to declare universally applicable minimum factors of safety.

Calculated minimum safety factors used by many engineers and organizations are listed in Table 3-4. These are presented herein for guidance only. The reviewer must establish minimum requirements based on site-specific conditions and his best judgment.

Users of this volume of the small hydropower guide manual should obtain copies of the referenced literature and technical reports for advice on the analytical procedures and criteria contained therein while investigating the integrity of an impoundment. Many other excellent and widely recognized publications, organizational or otherwise, are equally suited for those purposes.

TABLE 3-4
CALCULATED MINIMUM SAFETY FACTORS[a]

Case	Loading	Slope	Minimum Factor of Safety
I	Steady seepage, reservoir at normal pool	Downstream	1.5
II	Drawdown from normal to minimum pool elevations	Upstream	1.2
III	Earthquake		
	a. Case I with seismic loading	Downstream	1.0
	b. Reservoir at intermediate pool with seismic loading	Upstream	1.0

[a] Source: COE, 1977.

Stage 2 - Development and Evaluation of Data

General. As discussed previously, all available records, visual site examinations, and numerical engineering analyses that can be made with the available data are fully exploited in an effort to reach a dependable evaluation of impoundment integrity. Often additional data will be needed to augment the Stage 1 investigation before a dependable evaluation is possible, especially if the impoundment promises to be favorable for small hydro use.

The type of information and numerical data needed will concern structural, geological, and performance features unobtainable by direct visual examination. Some kind of exploration will be required for sample extraction; for providing access for direct observation; and for instrumental measurements of forces, stresses, deformation, seepage, etc. Data may also be obtained by non-destructive testing. Laboratory tests will be required to determine engineering properties of the materials of the dam and appurtenances and of the foundation for use in analyses and to assess their state of preservation.

The kinds of data, the techniques for acquiring them, and their applications in the integrity evaluation are discussed in this section.

Subsurface Exploration. The integrity of facilities may be questioned if foundation or embankment conditions are unclear, or if saturation levels and seepage levels are of concern. In such cases, subsurface exploration will be required to develop additional data and to provide samples for laboratory testing to determine engineering properties. There are many exploration tools and techniques available to obtain and develop data in the evaluation of existing impoundment structures. Some of the commonly used exploration tools are described below.

Geologic Mapping. The geologic map and geologic cross-sections are essential tools for planning a subsurface exploration program, specifically in evaluating foundation conditions of the impoundment under investigation. If geologic maps are available, they should be updated to show existing features such as slope instability, ground water seeps, etc., adjoining the impoundment and appurtenant works, as well as within the reservoir area.

Drilling. Information which can be obtained from drilling will be required for earth and rock dams if the original site conditions, design criteria and analyses, and construction records are unavailable or if visual inspection or performance records indicate that the facilities may not be performing adequately. The purpose of drilling is to obtain subsurface information which is used to construct a three-dimensional picture. Samples at depth can be secured, down-hole testing can be performed, water levels can be determined, and instrumentation such as piezometers and slope indicator casings can be installed. A large number of drilling and sampling systems are available to achieve the above purposes. Factors affecting the type of drilling and sampling used include type of impoundment structure, the materials constituting the embankment, abutments and foundation, and accessibility.

Core drilling with diamond drill equipment is the exploration method used most commonly for concrete or masonry structures and for relatively hard bedrock foundations. The core drilling program provides means of investigating and evaluating the structure and its foundation, construction joints, and cracking (if any) in the concrete or masonry. It also provides core samples for laboratory testing. The current practice of core drilling uses the rotary method almost exclusively because of the higher quality of samples obtained. The core barrel and diamond drill bit constitute the sampling device in which the cylinder or core of the sampled material is retained. Core barrels are available in a variety of sizes that produce cores with nominal diameters ranging from a fraction of an inch up to 48-inch or larger. However, NX-size cores (nominal diameter 2-1/8-inch) appear to be the minimum size core that would be meaningful for strength testing and visual inspection. In coring, the primary objective is maximum percent core recovery, so that the maximum amount of subsurface information is obtained. Unlike mining exploration, where the objective is almost solely maximum core recovery, drilling exploration for engineering evaluations requires that all available data during the drilling operations be collected and recorded. These data are of two types: permanent and fugitive. Permanent data are the cores obtained. Fugitive data are those which, if not observed and recorded in the hole log by the field geologist during the drilling operation, are lost forever. They include the time necessary to cut the core, the actions of the drill with the depths at which they occur, the driller's opinions, changes in the operation of the drill made by the driller with the reasons therefor, the color of the drilling water return, drill fluid "take" by the hole, and any other data of similar nature that may be requested by the project geologist. These data (which are not always obtainable from the drill core) are used to evaluate cracking, jointing, and other aspects of the material being sampled. It is the "non-core" information that may be most critical. Thus, it is most important that the geologist be at the drill and be carefully observing the details of the drilling operation at all times when the drill crew has the core barrel in the hole.

Core drilling is basically a sampling procedure for hard materials. In earth embankments or soil foundations the sampling procedure is entirely different, although the method of advancing the drill hole may be the same. The most commonly used method of drilling exploration in earth embankments is the straight rotary drilling method. Other drilling systems (such as augering) may be applicable, but they have certain limitations such as shallow depths, inability to utilize larger sampling devices (in the case of the hollow-stem auger), and difficulty in sampling below the water table. Percussion drilling is commonly used in alluvium containing cobbles and boulders. For many applications rotary drill rigs have several advantages. They can drill to greater depths than can be reached by other methods; they are extremely versatile; and they can accommodate different types of soil and rock samplers. Rotary drills conventionally use a circulating fluid (air, water or bentonite slurry) which is used to cool the cutting bit and remove cuttings by carrying them upwards to the surface.

Sampling, logging, and groundwater observation are the prime objectives of nearly all exploratory drilling. Sampling is essential for detailed examination and laboratory testing. Every precaution should be taken to guarantee that representative and uncontaminated samples are recovered. Basically, there are two types of soil samples: disturbed and undisturbed. Both types can be obtained using a variety of mechanical sampling devices. Sample tubes or barrels may be advanced into the soil by three basic methods: pushing, driving, or drilling. Pushing is usually preferred; however, in firmer material it often becomes necessary to drive or drill the sampler into the ground.

The most common type of sampler for obtaining disturbed samples is the split spoon. The split spoon is available in various sizes; however, the 1-1/2-inch-diameter sampler is popular because of correlations that have been developed between the number of blows required to drive the sampler into the soil strata and the relative density of cohesionless soils or the shear strength of cohesive soils. The sample obtained can be used for identification tests such as visual classification, water content, grain size analysis, Atterberg limit tests, etc.

Undisturbed samples preserve as closely as possible the natural structure and density of the in-situ material and are therefore suitable for strength tests as well as the identification tests that can be performed on disturbed samples. The open, thin-wall (Shelby) tube sampler is the most commonly used undisturbed sampler. The thin-wall tubes are pushed by the hydraulic or screw-fed system of the drill rig and are primarily used for sampling soft to stiff cohesive soils. The Shelby tubes are available in various diameters and lengths, but the most commonly used are 2 and 3 inches in diameter and 24 and 36 inches long. In embankment materials with gravel components, larger diameter (up to 6-inch) tubes should be used. A modification of the open thin-wall tube sampler is the closed-tube sampler in which a piston located at the lower end of the thin-wall tube is either released or withdrawn when the drive is started. The thin-wall stationary piston sampler and the Osterberg Piston sampler are examples of this type of sampler, which is used for sampling soft to stiff cohesive soils. The piston prevents shavings and "cave" material from entering the sampling tube and creates a partial vacuum between the piston and the sample which helps collect and retain the sample. When drilling with water or drilling below the groundwater table, the piston sampler offers the potential of more representative moisture contents and less contamination of the sample, because the sample tube is relatively dry when it reaches the bottom of the hole. When the material to be sampled is too soft to be cored or too hard to be sampled by pushing a thin-wall tube, modified push drill samplers are used. The Denison and Pitcher samplers are examples of this type. This type of sampler is primarily used for stiff to hard cohesive soils and dense cohesionless soils, and alternating hard and soft layers. They differ from the double-tube core barrel used for rock coring in that the stationary inner barrel (a thin-wall tube) extends ahead of the bit, thus preventing washing of weak materials. In the case of the Pitcher sampler, the inner tube is spring mounted; thus, the lead distance ahead of the bit depends upon the firmness of the material being sampled.

The U.S. Bureau of Reclamation's *Earth Manual* (USBR, 1974/3) contains a more detailed and comprehensive discussion of drilling and sampling equipment and should be used as a basic reference. Another good reference is *Basic Procedures for Soil Sampling and Core Drilling* (Acker, 1974).

Trenching or Test Pitting. These methods of exploration open a wider area of shallow subsurface materials to detailed examination than does drilling. The excavation can be done by backhoe or bulldozer or by hand. Inplace field density tests and disturbed or undisturbed samples can be obtained from the exploratory trenches or test pits. Undisturbed samples could be handcarved either in blocks, in random shapes, or into sample tubes. Trenches which are deep enough for a person to be buried if a wall were to cave should not be entered unless the sides are determined to be stable naturally or there is adequate wall support provided. All normal safety precautions and regulations should be observed.

Other Surveys. Ambient vibration surveys on concrete or masonry structures measure the natural mode of vibration, and their shapes and periods of vibration.

Underwater surveys can be conducted to visually evaluate physical conditions of upstream earth embankment slopes or concrete dams under water. For example, concrete or masonry dams can be inspected for deterioration or cracking and earth embankment slopes for slides or deformation of the slope.

Laboratory Testing. Laboratory test results are performed to obtain data which is used for both rational evaluation of conditions and to obtain numerical data for use in engineering analyses. For convenience, laboratory tests are divided here into two categories (1) soils and (2) rock and concrete. All testing should be performed at established laboratories by experienced personnel. Therefore, only the types of test, their purposes, and the use of test results are discussed herein; test procedures are beyond the scope of this work. Test descriptions and procedures are available in several sources, with the *Annual Book of ASTM Standards* (ASTM, Annual) as the best of these. Parts 10 and 11 of the ASTM Standards cover concrete and soils respectively.

Soils. Laboratory testing of soils and soft rock consists of two types of test - (1) classification and physical properties testing and (2) engineering properties testing.

The Unified Soil Classification System is the system most commonly used for classifying soils. This system is based on a recognition of the various types and significant distribution of soil constituents, considering gradation characteristics, and plasticity of materials. Grain size distribution data and the results of Atterberg limits tests provide the information, except for the determination of organic content, to properly classify the material. The USBR's *Design of Small Dams*, (USBR, 1974/2), describes the Unified Soil Classification System and presents general properties of materials for each of the soil classification groups.

Other tests commonly performed which are not truly classification tests or engineering properties tests include natural water content, dry density, and specific gravity.

Engineering properties can be roughly estimated by experienced soil engineers if the soil classification is known. The estimated engineering properties may be adequate for preliminary evaluations or if the structure is obviously adequate. However, if detailed engineering analyses are to be performed, the engineering properties must be determined by laboratory tests. Tests that are commonly performed to determine engineering properties of soils include compaction tests to determine the moisture-density relationships of materials containing a significant percentage of fines; relative density tests to determine the maximum and minimum densities for relatively clean sands and gravels; consolidation tests; permeability tests; and shear strength tests. The engineering properties tests listed above are relatively straight forward except for the shear strength tests. Shear tests can be performed in direct shear apparatus or in triaxial shear apparatus. Normally all shear tests will be performed on saturated samples. In the direct shear apparatus, reliable pore pressure measurements cannot be made; the only pore pressure control available is to run the test slowly enough for pore pressures to dissipate, or rapidly enough so that the pore pressures build up in the sample to simulate field conditions where high pore pressures are expected to exist. Triaxial tests afford the opportunity to make good pore pressure measurements and the results from one triaxial test series can be utilized to determine both effective and total stress strengths or, in the terminology of the U.S. Army Corps of Engineers, "S" and "R" strengths respectively (S standing for slow rate of failure in the direct shear apparatus where pore pressures are allowed to dissipate, and R standing for rapid failure where pore pressures are not allowed to dissipate).

All engineering properties tests on materials from existing embankment and foundation materials that are to be left in-place should, if possible, be performed on undisturbed samples. The sample size should be large enough to permit testing of representative samples without having individual particles control the test results.

Concrete and Rock. Tests on concrete and hard rock samples are normally limited to determining the unconfined compressive strength. A standard method of determining the unconfined compressive strength of rock is contained in the *Annual Book of ASTM Standards,* Part 11. The method presented for unconfined compressive strength of rock core specimens is also applicable for concrete core specimens.

Analyses and Interpretations of Results.

The final numerical analyses and the methods employed in Stage 2 are the same as those described and enumerated by reference above in the discussion of Stage 1 - Methods of Analyses. However, they are more extensive, definitive, and refined. They are specifically tailored to represent the actual physical conditions disclosed by the investigations. Particular care should be taken to study suspicious or uncertain appearing features and conditions. The engineering data and information to be used in the analyses are those specifically obtained for that purpose during Stage 2. For example, if the spillway capacity appears inadequate for any reason, such as experienced near capacity discharges or high regional flood comparisons, a new flood estimate should be made and the existing spillway and impoundment components should be analytically tested for their ability to safely handle the updated flood. Or, for example, if the stability of an embankment-type dam appears marginal for any reason (such as apparently over-steep slopes, unusual saturation patterns, low strength soils, or indications of high foundation pore pressures) a stability analysis and companion seepage analyses should be made using soil strengths and permeability rates obtained by sampling and testing for use in those specific analyses.

In many cases, the final analyses will be the only analyses, rather than extensions and refinements of Stage 1 analyses.

As valuable as they are, numerical analyses cannot provide total and absolute answers upon which to base the final evaluation. Many physical conditions and reactive mechanisms cannot be mathematically analyzed, even qualitatively.

When all the objective factors that may influence the evaluation have been gathered, interpreted, analyzed, and discussed, the investigator must decide if the impoundment can be safely used to serve a small hydropower installation in its present condition, that it cannot, or that it is engineeringly feasible to rehabilitate it so that it can.

There are no clear-cut rules by which these decisions can be made. Instead the decisions are made by a value judgement process employing empirical reasoning and objective assessments by trained engineers. Comparisons with successfully performing similar impoundments are made. Criteria generally accepted and proclaimed by reputable practitioners and by professional engineering societies are applied as general tests in measuring adequacy. Throughout the decisions process, the general classes of concern enumerated for site inspections must dominate the mind of the evaluator.

The type, size, complexity and regional setting of impoundments are highly variable. For that reason, Exhibit II, "Considerations and Procedures for Impoundment Integrity Evaluations", is included at the end of this volume to provide a comprehensive list of actions, studies, and reviews that constitute the evaluation process. Obviously, all items are not applicable to all impoundments, nor are all the items of equal importance.

Stage 3 - Rehabilitation Methods and Cost Estimate

When an evaluation decision finds that an existing impoundment is suitable for a small hydropower installation, it will be possible to proceed directly with the feasibility study described in Volume I, provided no deficiencies were disclosed by the integrity investigation. However, it can be expected in some cases that the investigation will identify structural or hydraulic weaknesses in the dam or appurtenances, or even the reservoir confines, which would require remedial treatment before the impoundment could be safely used for a hydropower installation, whether or not such installation is close-coupled to the impoundment. In such cases, it would be necessary to formulate repair or alteration schemes for rehabilitating the particular component of the impoundment and to estimate the associated construction costs.

The required repairs or remedial measures may be simple or extensive and their costs will vary accordingly. Alternative designs and construction procedures are often feasible and their physical and cost advantages should not be overlooked. It may be possible to combine or coordinate the rehabilitation repairs or remedial measures with any alterations that might be needed to accommodate the hydropower installation. The need to maintain stream flows or continue operation for existing project purposes during repair must be considered and in some cases may control or influence the design and the construction schedule for the repairs.

Some of the deficiencies most likely to be encountered and examples of corrective repairs and reconstruction are presented and discussed in Section 4. The associated cost estimating procedures and the use of the cost estimates in making decisions regarding rehabilitation of the impoundment are presented and discussed in Section 5.

Program Administration and Personnel

General. As discussed in Section 1, it is imperative that the integrity of the impoundment be positively established because of the potential for capital investment loss and public liability should the dam fail. The feasibility of the power project depends on a sound dam or one that can economically be made sound. The integrity investigation must be conducted in a comprehensive, orderly manner by an individual or team educated and experienced in several technical and scientific disciplines essential to dam engineering, construction, and operation. The team will function most effectively if it is properly structured and managed to accomplish specific objectives on an established schedule. The size of the team and the different disciplines required will vary with the type and complexity of the impoundment and the breadth of each individual's expertise. The teamcomposition may also vary somewhat with the particular stage of the investigation.

Establishing and Administrating an Investigative Program. The organization and direction of the integrity investigation program should be assigned to an engineering program manager who has broad and extensive experience in design and construction. The program manager should be an engineer, usually in the field of general civil engineering. Non-technical personnel should not be assigned as program manager.

An initial schedule should be established for the overall program with each of the three stages separately identified. Target dates for the fundamental decisions of each stage should be established, while recognizing that the investigation may be terminated at the end of any stage or that Stage 2 or even Stage 3 might not be required, as previously discussed. The schedule should recognize and provide for sequential or simultaneous conduct of activity. For example, the site inspection should not precede the acquiring of existing data because familiarization with that data will provide special guidance for the inspection. Where data are to be acquired in Stage 2, other Stage 2 analyses independent of that data can proceed simultaneously.

The schedule must provide for flexibility so that as the objective of each stage nears achievement and the initial and final integrity decisions are made, the next appropriate activity can proceed without delay. A sample schedule for a relatively simple investigation is presented below. Additional inspections for specific or more detailed observations will be advisable as the investigation proceeds and schedule allowances should be made for that purpose.

Sample Schedule

Stage 1: June 1 - August 5	
Collect and evaluate available data	1 - 2 weeks
Site inspection	1 week
Evaluate integrity of existing facilities, develop Stage 2 program (if required), and prepare report	1 - 2 weeks
Stage 2: August 10 - October 31	
Administration and coordination	1 week
Subsurface exploration	1 - 2 weeks
Laboratory testing	1 - 2 weeks
Evaluate exploration and laboratory data, perform engineering analyses and evaluations	2 - 3 weeks
Stage 3: October 15 - November 30	
Prepare rehabilitation design	1 - 2 weeks
Compute construction quantities	1 week
Prepare construction cost estimates	1 week

If, during Stage 1, it becomes apparent that Stage 2 will be necessary, the specific drilling, sampling, and testing objectives and procedures must be planned in detail and their manner of accomplishment decided. Time requirements, costs, scheduling, and instructive procedures must be considered and established. If the services are to be provided by others, service agreements or contracts must be arranged. Rights of entry may be necessary. The dam owner's permission and liability clearances must be negotiated and obtained. The exploration must be coordinated with existing project operation schedules or requirements. Jurisdictional authorities may have legal controls that must be satisfied.

If more than one impoundment is under investigation by the group, management may have to establish priorities. Management should also recognize any advantages in staff utilization by coordination of activities for a multi-project program.

Should a difficult, complex, or unusual engineering problem arise, it may be advisable for the investigating group to retain a consultant or individual expert for advice, and management must arrange for those services. Such need might occur, for example, while deciding upon the manner of exploration or test of a suspected unsafe foundation condition during Stage 2. Or advice might be needed on the magnitude and severity of expected earthquake ground motions and foundation displacements at the site for use in studying the dynamic stability of the dam during Stage 2.

As Stage 2 nears completion and it is decided that Stage 3 is in order, management must schedule the study of rehabilitation methods and preparation of cost estimates. The study should provide for alternative plans to determine the possibilities of cost advantages.

Personnel Qualifications and Composition of Investigative Team. The minimum integrity investigation will include records review, site examination, and judgemental evaluations. Since evaluations at this stage are largely judgmental, it is important that an individual or group experienced in all phases of dam engineering perform the investigation. As a minimum, the individual or group must have scientific knowledge and experience in the fields of geotechnical engineering, structural engineering, and hydrology and hydraulics as related to water retention structures.

When Stage 2 and Stage 3 are to be performed and the investigations are relatively straight forward, the individual or group that performed the Stage 1 investigation can perform the additional work with support from lower level staff. The amount of work shown in the sample schedule represents a relatively simple Stage 2 and Stage 3 investigation that could be performed by an individual or small group, except for the drilling and laboratory work, which requires special equipment. This type of program would cost in the order of $15,000 at 1978 prices. If the Stage 1 investigation reveals questionable integrity such as the need to perform a complete seismic analysis of an earth dam, additional expertise and substantial costs (in the order of $100,000 at 1978 prices) will be required. This type of investigation would economically be practical if the anticipated revenue is high for a small hydroelectric project but would not be justified if the project was considered to be economically marginal.

Peer Review. Evaluation decisions seldom can be based solely on the results of mathematical analyses or simply on the external appearances seen at the time of the site examination. Decisions are made mainly by empirical analyses and judgmental evaluation, supplemented by the mathematical analyses and site examination.

The report of the Los Angeles County Coroner's Jury after the failure of St. Francis Dam in 1928 notes: "...public safety... demands that the construction and operation of a ...dam should never be left to the sole judgment of one man, no matter how eminent, without check by independent expert authority, for no one is free from error, and checking by independent experts will eliminate the effect of human error and ensure safety."

The statements in the two preceding paragraphs emphasize the reasons for and the purposes of peer review, especially when evaluations are being made in difficult or unusual circumstances. The wise investigator will recognize when and why he should seek peer review. Peer review is available from individual consultants and from other engineering firms engaged in dam design.

SECTION 4
REHABILITATION METHODS

General

The integrity evaluation will find that an impoundment is (1) safe in its present state; (2) unsafe and obviously cannot be rehabilitated economically for hydropower use (in which event removal of the dam by the owner would seem to be in order under some dam safety regulatory process); or (3) defective in some manner, but may be restorable economically for hydropower use.

This section discusses various ways in which defective dams have been successfully restored and used for the safe storage and control of water. Engineering feasibility is emphasized. The possible alternative solutions, considered with their costs, can then be incorporated into the feasibility study.

Most defects in an existing dam and in the appurtenances are usually associated in some way with one or more of the following physical circumstances:

1. Reaction of the foundation formation and construction materials to their environment.
2. Resistance to forces and loads
3. Control of seepage
4. Hydraulic capacity and flow performance characteristics
5. Serviceability of mechanical/electrical components and systems.

Many general examples will immediately come to mind, e.g., alkali-aggregate reaction in concrete structures in the case of (1); slides in earth embankments for (2); emerging seepage under pressure from drains in concrete dams or along the toe of an earth dam for (3); stream bed erosion and undercutting of a spillway terminal structure for (4) combined with (2); seizing of an outlet slide gate or a neglectful dismantling of a spillway radial gate hoisting system for (5).

The defects arise either because of original poor design, shoddy construction, lack of maintenance, changed operational demands, or from the application to the original design of more dependable, present-day analytical methods and accumulated hydrologic and seismic records.

The defect may be extensive and seriously threaten the structural integrity of the dam unless promptly counteracted by extensive repair or even replacement. The defect may be in an early stage of development, and if so can be successfully arrested by intensified maintenance. The true nature of a suspected defect may not be immediately determinable and a period of operational monitoring instrumentally or visually may be needed for diagnosis. It may be possible to eliminate or mitigate the defect by reducing the storage level permanently or by operating the reservoir in a different manner.

Decisions on alternatives are thus influenced not only by differing physical designs and methods but also by differing funding arrangements. An extensive replacement such as a new, relocated spillway to replace one historically threatened by obstruction from slides, ice formation, or drift accumulation requires a capital investment. Alternatively, an improved, more attentive maintenance program for continual patrolling and removal of obstructions would require increased annual maintenance funding. Or the useful remaining life and cost of a repair such as patching rotted portions of a timber facing of a rockfill dam might be compared with the life and cost of total removal and replacement with a reinforced gunite facing. The reduced benefits resulting from operating the reservoir at a lower stage for increased flood detention capacity might be compared with the capital cost of enlarging the spillway discharge capability.

Rehabilitation of Dams

In this section and the one that follows the rehabilitation of dams and their foundations are discussed separately. However, it cannot be emphasized too strongly that a dam and its foundation must perform together as an integral unit. This is especially significant along the immediate interface. Many defects simultaneously implicate the dam and the foundation, especially in the case of embankment dams.

Earth and Rockfill, Stonewall-Earth, and Rockfilled Timber Crib Dams. The more common defects encountered are:

1. Insufficient control of seepage and of the accompanying pore pressures and escape gradients.
2. Overly steep slopes of marginal stability, incipient slides, loss of freeboard from crest settlement.
3. Severe erosion and benching of the upstream slope, deep gullying of the downstream face and groins—all tending to reduce the embankment cross section at the most critical elevations.
4. Transverse cracking of the embankment from differential settlement of the fill and consolidation upon saturation of the foundation.
5. Crushing, cracking, parting of waterstops in concrete face slabs of rockfill dams from settlement and deformations of the fill.
6. Excessive large tree growth with large root systems near or on the dam crest creating a breaching potential from uprooting during high winds or root deterioration after the tree dies. Rodent holes can cause similar problems.

7. Utility pressure conduits penetrating or traversing the dam.

Examples of successful remedial measures for these defects are described in the same order. The reader must recognize that in every case the specific details will be different and that the construction methods must be adapted to the actual conditions.

1. Seepage through so-called homogeneous earth dams, where permeability is relatively high or where leakage may concentrate through anomalous regions or transverse cracks, can be controlled by placing a compacted, more impervious zone on the stripped face of the existing dam. The reservoir must be emptied. If the normal drawdown operation of the reservoir cannot be limited or if the new slope cannot be made sufficiently flat, a pervious zone surmounting the added impervious zone may be necessary. If the defect includes excessive seepage through the foundation or along the interface with the dam (often the result of inadequate foundation preparation originally) the new impervious zone can be extended into a cutoff trench excavated into the bedrock formation across the valley section and into the abutments along the upstream toe of the dam or upstream as a blanket. Time must be allowed for accumulated silt deposits to dry; or excavating by dragline may be possible.

If seepage emerges uncontrolled along the toe or over the lower portion of the downstream face, a berm or mildly sloping zone of sand and gravel or cobbles and rock fragments may be added to that face. The grading of the materials positioned immediately against the dam and abutment hillsides must be much more pervious than the material upstream and also prevent movement (piping) of fines from the dam or foundation. If pervious material of the requisite grading is scarce or costly, the main body of the added mass can be comprised of other types of materials, if they are enveloped by pervious materials at all interfaces. With variations, this treatment also improves downstream slope stability.

If the seepage is largely concentrated along the toe or groins, a drain pipe of clay tile, sewer tile, or asbestos-bonded CMP, successively enveloped by gravel and by sand, can be installed in a trench excavated into the foundation along the toe of the dam. If the drain can be safely installed on an alignment upstream of the toe, it will be more effective, especially for slope stability.

2. Actual slope failures can be repaired by first removing all or critical portions of the disturbed mass. If the strength of the materials within the mass has been permanently reduced or if the internal deformations adversely affect the function of a particular zone, then reconstructing to new configurations and zoning suited to the engineering properties of the construction materials is called for. The materials used in reconstruction may be either derived from new sources or reused from the slide volume.

Upstream slope failures are most likely to result from drawdown. Reconstruction requires lowering or even emptying the reservoir. The configuration of the slope to be reconstructed is established by analysis using the engineering properties of the available materials and applying the proposed reservoir operation plan. Construction of a drawdown zone of free-draining rock or cobbles should be considered.

If slide movement has not actually occurred but is considered possible, the slopes can be strengthened by various combinations of seepage control for reduction of destabilizing pore pressures and by adjustments of the exterior slopes. Slopes may be flattened or bermed in lower elevations or unweighted in upper elevations. Free-draining buttress or reverse filter blankets can be added over the ground beyond the toes to counteract instability from high pore pressures in confined, buried aquifers.

The design elevation of the dam crest can be restored by simply stripping the surface and placing and compacting more soil on it. If the crest is narrow, local steepening of both slopes may be acceptable for accommodating the restored elevation, or even for an increased elevation when greater freeboard is needed. Reinforced concrete parapet walls can also be used for either restoring or increasing freeboard.

The near-vertical downstream face of a stonewall-earth dam can be strengthened by adding a downstream zone of compacted, free-draining rock on a slope somewhat flatter than the natural angle of repose of the added rock. The filtering capability at the original interface between the upstream earth zone and the rock wall must be carefully investigated. If piping has occurred, or is likely to occur, a properly graded transition zone should be placed between the existing rock wall and the added rock. The transition zone must be terminated in a non-pipable formation across the channel section and up the abutments, so that all seepage is forced to pass through the filter. Sink holes in the earth zone can sometimes be excavated, shaped, and backfilled with filter materials and compacted earth, and a new compacted earth zone placed on the existing upstream slope to improve the long-term suitability of the impervious zone. If indications of piping, sink holes, and slope disruptions are extreme, rehabilitation by these methods may be inadequate.

Extensive restoration of decayed timber elements of a rock-filled timber crib dam is generally not feasible. Depending upon the degree of disruption and the quality of the rock originally retained by the crib, it may be possible to rehabilitate the dam by adding transition and filter zones and an impervious earth zone upstream and utilizing the old dam as a downstream shell element.

3. Upstream slopes severely benched by erosion can be restored by surface stripping and replacement with

compacted fill. A cushion or transition bedding of correctly graded sand and gravel or small rock is placed on the restored slope beneath the riprap stone. This bedding is essential for adequate performance of the slope protection. Soil-cement properly proportioned, placed, and compacted has been used to restore a slope and to protect it against wave action at the same time.

Gullying of the downstream face and groins can be mended and recurrence prevented by excavating to provide working room, refilling the eroded and excavated areas, then placing a protective course of crushed or angular rock. A system of concrete surface drains, cast or preformed, installed on narrow berms and coupled with a nurtured cover of local grasses has also been successful.

4. Transverse cracking can be repaired if the causative forces have stabilized or have attenuated with time. One method has been discussed in (1) above. When the cracks are limited to the higher elevations in the dam, as they usually are, a narrow trench can be excavated from the dam crest and backfilled with impervious plastic soils. The reservoir may have to be drawn down or even emptied during repair. The strength of the backfill materials must be adequate, otherwise a critical failure plane may be induced by the backfilled trench. Reinforced plastic fabrics, anchored or planted along their perimeters, placed on a smooth prepared surface on the upstream slope and covered by a protective element, can be considered.

5. Excessive leakage caused by disruption of the concrete face elements of a rockfill dam can be reduced or eliminated by selective removal and replacement of damaged panels, if the waterstops from adjacent panels are serviceable. If the embankment is still settling at a significant rate, the repair process will have to be repeated several times. The damaged panels can be covered with courses of redwood tongue and groove planking for increased flexibility during the active settlement period. Anchored butyl rubber sheets have been successfully used on the surface of the panels to waterstop the panel joints.

A rockfill dam can be modified to include an inclined earth core by using the existing dam for the downstream shell and constructing transition zones, filter zones, impervious zones, and shell elements upstream. The opportunity for improved control of foundation seepage, if necessary, is available in such an alteration.

6. The upper crest sections of embankments that are riddled with tree roots or rodent holes can be restored by complete removal of the infested portions and by replacement with compacted fill securely bonded to the unaffected portions.

7. A utility pressure conduit located longitudinally on or near the dam crest can be totally relocated, or it can be rerouted at normal pool level on the upstream face if the reservoir is usually operated full. A longitudinal or transverse conduit can be totally encircled by a larger diameter pipe, or partially encircled underneath by a semi-circular pipe segment of sufficient capacity to safely transport water from a ruptured conduit away from the dam. Automatic shutoff valves controlled by pressure sensing devices can be installed in the conduit beyond both ends of the dam. Transverse conduits can be either relocated away from the dam or replaced using the proven design principles and upstream gating arrangements that are employed for safe outlets.

Concrete and Masonry Dams. The more common defects encountered are:

1. Concrete deterioration from alkali-aggregate reaction, frost action, and poor concrete and construction methods originally.
2. Excessive uplift on the base, on foundation planes at depth, and on horizontal construction joints.
3. Marginal stability for reasons other than excessive uplift.
4. Overstressing, especially in buttress type dams.

Successful remedies and repairs are discussed in the same order.

1. Concrete deterioration appears to be the most prevalent concrete dam defect. The great advances in cement and concrete technology and manufacture and in concrete placement methods are most likely responsible for the improved resistance to deterioration now being observed in newer dams, and it would be expected that this would be confirmed by future performance as the dams become older.

Alkali-aggregate reaction once started cannot be totally stopped by any means now known. If deterioration is advanced, the defective concrete can be removed. For example, in an arch dam if the concrete is less severely affected at lower elevations, its useful and safe service life can be extended at a reduced storage capacity by removing the upper portions and converting the lowered crest to an overpour spillway. The defective concrete can also be replaced. If the entire dam is badly deteriorated but the reservoir basin, detached appurtenances, available yield, and power head provide sufficient benefits, a new dam can be constructed in close proximity to the existing dam or even on the same site by removing the old dam. If the site topography is suitable, the old dam can even be incorporated into a new embankment-type dam.

Alkali-aggregate reaction can be slowed and the useful life of the dam extended by the application of protective upstream coatings and by densifying the concrete itself by grouting, all in order to reduce the severity of the wet environment which helps promote the reaction.

Deterioration of an upstream dam face from alternating freezing and thawing action can be repaired by scaling and chipping the surface to fresh concrete. Steel forms or precast concrete panels can be positioned to the restored face configuration and the intervening space filled by the preplaced aggregate concrete process. Once the panels or forms are installed, the repair can be completed with water in the reservoir. This method restores the full dam cross section. Gunite or shotcrete directly applied with the reservoir empty, of course, can be used if the dam cross section has not been diminished significantly. Seal coats of materials such as neoprene rubber compounds and asbestile can be applied to the prepared concrete surface.

2. Excessive uplift results from inadequate control of seepage. If there are foundation drains and formed drains in the dam which have become plugged with chemical deposits, they can sometimes be reamed and their effectiveness restored if they are accessible from drain galleries or from the dam crest. New foundation drains can be drilled. If water losses are excessive, the foundation can be regrouted from galleries, if they exist but the more effective way to reduce uplift is the addition of drainage.

3. Marginal or inadequate stability in a concrete gravity dam can be counteracted by installing post-tensioned stress tendons through the concrete section and into the foundation. The resisting capabilities of gravity thrust blocks for an arch dam can be increased by the addition of concrete or by post tensioning into the foundation. Post tensioning of a gravity section is especially suited where the horizontal lift surfaces cannot transmit shear because they were not cleaned of laitance during construction.

The stability of a gravity dam can be increased by building concrete buttresses against and bonded to the downstream face. Reservoir water load during construction, temperature control of the new concrete, preparation of the old weathered concrete surfaces, and details of the joint between the two require special design considerations and construction sequences for proper transmittal of shearing stresses and achievement of load sharing. Stress magnitude and distribution, as well as stability, can be improved in both gravity and single arch dams by increasing the cross section with added mass concrete downstream. Slots are left between the new and old concrete for later filling when the new concrete temperatures have equalized.

Stability of buttress-type dams is discussed under (4) below.

4. Buttress-type dams most likely to be encountered during small hydropower feasibility studies are concrete multiple arch dams and concrete or timber slab and buttress dams. Dams having concrete buttresses and removable timber flashboards may also be found.

Sliding stability will seldom be a problem if the angle between the upstream face and a vertical plane is substantial. Because of historically changing construction costs, most of the buttress dams of concrete will be quite old, 50 years or more. Consequently, defects will not only be associated with the inherent low quality and deterioration of vintage concrete but also with stresses in the members comprising the dam. Characteristically, very little reinforcing steel was used in these older dams.

High tensile stresses in the upstream regions of the buttresses of a dam can be reduced by installing tendons or high strength steel rods along the groin at the face of each buttress between the arch barrels, anchoring them into the foundation and then stressing them a predetermined amount while the reservoir is at a low stage. The tendons are covered with protective concrete. The same technique can be used along the intrados of the arch barrels on both sides of the buttress. Lateral rigidity of an individual unreinforced buttress can be increased by attaching reinforced bond beams on both sides of the buttress or by attaching vertical pilasters.

Indicated high stresses in arch barrels attributable to loss of effective thickness from concrete deterioration can be remedied by scaling and chipping the extradosal surface and then restoring, or even increasing, the thickness with gunite or shotcrete reinforced with steel bars or mesh.

Cross channel stability during earthquakes may be low or lacking. The arch barrels and architectural struts between buttresses supply very little resistance. This defect can be overcome by converting alternate pairs of buttresses into single, tower-like supports. This can be done by adding a series of steel or reinforced concrete truss members or vertical concrete diaphragms between the two buttresses. The joint details are extremely important for safe load transfer, especially if the existing buttresses are only nominally reinforced. The buttress can be stiffened by bond beams or pilasters.

Defects in timber buttresses and decks are mainly associated with rotting, corrosion, or other deterioration of the materials forming the members and joint fasteners. Reconstruction with new materials must be undertaken.

Rehabilitation of Dam Foundations

The importance and consequences of foundation defects will vary with the type of dam and the degree and methods of rehabilitation must be planned accordingly. For example, a geological defect such as an open joint at the surface of a rock foundation beneath an embankment dam is of much greater concern than it is beneath a concrete gravity dam. The physical features of a foundation defect usually are not directly observable, because they are hidden by the dam. The presence of characteristics of the defect must be deduced from indirect as well as direct evidence obtained instrumentally or from extracted cores and from study of visual manifestations, such as dissolved solids in seepage water, or movements and strains in the dam itself.

Foundation rehabilitation is often difficult and in some cases may not be possible.

Some of the more common defects encountered are:

1. Insufficient control of seepage and consequent piping, dissolution, or softening of the foundation materials; and displacement of rock masses.
2. Insufficient supporting strength.
3. Inelastic deformations.
4. Loss of local supporting capability from undercutting due to rock plucking.
5. Presence of faults.
6. Excessive or differential consolidation and subsidence.

Some remedial measures that have been used for these defects are described in the same order. Because the foundation must support the dam without excessive deformations or displacements in either the dam or foundation and must control seepage as well, it is obvious that the foundation defects which influence stress and stability in the dam and their rehabilitation cannot be considered independently of the dam.

1. Seepage through foundations can be controlled by grouting, blanketing, new cutoffs, drainage, and pressure relief wells.

A grout curtain can be installed beneath the impervious zone of an embankment dam by drilling through the dam. Care must be used to avoid hydraulic fracturing of susceptible fills with the drilling fluid. Injection of grout between the foundation surface and the base of the embankment should be done carefully. Different techniques are available. A new grout curtain can be installed or an existing curtain supplemented beneath an arch or gravity dam from existing foundation galleries, along the upstream toe, with the reservoir emptied, or even by drilling from the dam crest. A grout curtain can be installed beneath a thin arch dam by slant drilling from the downstream face.

An impervious blanket of compacted earth or a commercially available liner can be placed on the floor of the reservoir. The blanket must be joined to the impervious element of the dam and to the abutments, and must terminate in a satisfactory manner.

The construction of a new cutoff and an impervious facing is described under item (1) of the subsection "Earth and Rockfill, Stonewall-Earth, and Rockfilled Timber Crib Dams." A new cutoff can also be formed in alluvial deposits with a slurry wall. The wall must be joined to the impervious element of the dam. A horizontal impervious zone (blanket) can sometimes be used.

Embankment toe drains and drain blankets are described in the same subsection referenced above. The toe drain or part of the blanket drain can also be installed at depth in the foundation for dual service.

Pressure relief wells or trenches backfilled with drain rock and filter material can be drilled or excavated along or beyond the toe of an embankment dam to control the escape gradients of seepage flowing through the foundation.

Drain holes beneath gravity dams are described under item (2) in the subsection "Concrete and Masonry Dams."

Drain holes can be drilled along the downstream toe of an arch dam, greatly reducing the possiblity of high uplift pressures in the rock structure which tend to displace a foundation rock mass at the abutments.

A drain tunnel can be driven into the foundation from an abutment hillside at an embankment dam, or even from an existing gallery in a concrete dam.

2. The strength of a foundation beneath an existing dam is difficult to increase directly. Tensioned rock bolts or steel tendons may increase the strength of rock foundations, and consolidation grouting may increase the strength of sand and gravel foundations. However, the forces that must be resisted can be changed, or additional resistance can be provided. For example, the imposed shearing stresses on a weak clay seam or bed in a horizontally stratified sedimentary formation can be reduced by flattening the slopes of an embankment dam or by adding buttressing fills if the weak bed outcrops on an abutment hillside. Beneath a concrete dam, the resistance to sliding can be increased by casting concrete shear keys across the bed from trenches or drifts; but it is a difficult and expensive process.

Loose to medium-dense sandy alluvial foundations lose strength during prolonged ground motions from earthquakes. Increased drainage, consolidation of loose materials, and increased confining pressure would all improve the strength of the materials during earthquakes. However, drainage and consolidation may be difficult to achieve and the increase of confining pressure may result in additional dynamic stresses and may actually decrease the stability. The imposed shearing stresses are also difficult to reduce by exterior adjustments of the dam configuration. The most positive way of increasing the stability is to remove the susceptible soils in preparing the foundation beneath flattened slopes or buttressing zones.

3. Irrecoverable deformations in hard rock foundations, which are of concern primarily for concrete dams, occur on first loading, when the mass modulus of elasticity is lower than for subsequent loadings. For existing dams of the moderate sizes under consideration here, it may not be practical or even necessary to attempt treatment of the foundation if it can be demonstrated that the dam is not presently overstressed and that irrecoverable deformations are not continuing.

The deformation characteristics of limited masses of rock defined by geological structural features can be altered by a combination of consolidation grouting and tensioned rock bolts or steel tendons.

4. Local losses of hard-rock foundation may be caused by overpour along the downstream toe of gravity sections, along buttresses, and along the contact between the abutment and extrados of arches. The resulting cavity can be filled with concrete and the resulting interfaces between the rock foundation and the dam concrete then grouted after the concrete mass has cooled to ambient temperature. A plunge basin deeply eroded and retrogressing into the adjacent foundation of an arch dam can be unwatered, cleaned out, and covered with mass concrete anchored to the rock, coupled with treatment of the cavity beneath the dam as just described.

5. Treatment methods for inactive faults or large shear zones beneath existing dams are limited because they are not directly accessible. If a transverse-trending fault is transmitting seepage, it can be locally mined out to practical depths near the toes of the dam and plugged with concrete upstream and filled with filtered, free-draining materials downstream.

Active faults cannot be treated. Instead, the ability of the dam to accommodate fault displacements without disastrous release of water must be evaluated, and if necessary the dam must be modified to accommodate expected movements without failure.

6. Excessive or differential consolidation and subsidence cannot be effectively arrested or controlled by any direct treatment of the foundation at depth. Instead, any continuing foundation movements and their effect on the dam are continuously monitored. The dam can then be repaired or modified accordingly. In some cases, the cause of the subsidence may be detected and corrected, especially if the subsidence is related to old mining works or fluid withdrawal from the substratum beneath the dam.

Rehabilitation of Appurtenant Works

Spillways. The more common defects encountered are:

1. Inadequate capacity to safely pass floods without overtopping the dam.
2. Unpredictable capacity.
3. Damaging hydraulic performance characteristics caused by extreme channel convergence or curvature, lack or mislocation of energy-dissipating terminal structure, excessive velocities, shifting hydraulic control sections, etc.
4. Obstructions to flow.
5. Controlled spillways without redundant features for embankment dams.
6. Spillways founded on fill materials or located over embankment dams.
7. Structural weaknesses in channel walls and inverts, gate piers and anchorages, retaining walls, conduits, etc.
8. Poorly maintained or inoperative mechanical/electrical components.
9. Concrete deterioration.

There are many types and configurations of spillways. Locations vary extensively and are influenced by many factors related to the site and to the type of dam. Consequently, only a few examples of remedial measures can be included here.

1. It may be physically impractical to increase the capacity of a spillway—one with a tunnel discharge carrier, for example; but its capacity can be usually supplemented by a second spillway separately located.

An open channel spillway capacity can be increased by raising the dam crest in different ways, including a parapet wall, even on an embankment dam. Approach channel and discharge channel freeboard must be investigated. If necessary, they can be increased by extending the walls or linings in various manners. A weir type control structure can be lengthened if a new transition to the discharge channel can be fitted in structurally and hydraulically. Usually a capacity increase can be made more efficiently by increasing the head rather than the length, because the capacity varies with the three halves power of the head.

The ability of an impoundment to safely pass floods newly estimated at greater magnitudes can be achieved without enlarging the spillway, if increased flood detention storage capacity can be economically dedicated and the project scrupulously operated accordingly.

Indicated overtopping by the new flood for infrequent limited durations may be acceptable at a concrete dam on an erosion-resistant foundation.

The existing spillway can be considered a service spillway and a new so-called emergency spillway constructed at a higher elevation designed to operate only during a very infrequent flood of the largest magnitude. Project damage, especially to the emergency spillway, can be economically accepted.

Fuse plug control devices in spillways are unpredictable and can create peak flows greater than those of the natural flood. They may also fail to work and thus not provide the intended protection from the inflow flood.

2. The capacity of a siphon spillway may not be reliably predictable. It is also vulnerable to obstruction by trash and ice. It discharges sudden flows at high rates. A battery of siphons can be converted to an open free-discharge crest by removal of the siphon hoods and reshaping of the crests. If additional freeboard is needed with the modified crest, it can be provided as discussed in (1).

3. Freeboard can be increased for an open discharge channel by raising vertical sidewalls or extending a sloping lining to contain overtopping waves or rideups created by excessive channel convergence or alignment curvature. A sloping lining can be extended with a vertical wall. A curved channel can be compartmented by several vertical training walls which will decrease the

rise of the water surface on the outside concave wall in proportion to the number of compartments.

Ill performance of a stilling basin set too shallow can be improved by imposing sufficient tailwater with an end sill or downstream weir.

A foreshortened stilling basin can be extended to compensate for jump sweep-out.

Existing retrogressive channel erosion can be arrested by adding a bucketed terminal structure positioned well above tailwater and supported on deep-seated, cast-in-place piling in drilled holes.

4. An incipient slide or overly steep slope endangering a spillway approach or discharge channel can be stabilized by methods similar to those discussed in the subsection "Earth and Rockfill, Stonewall-Earth, and Rock- filled Timber Crib Dams," item (2).

Persistent drift and trash can be held at bay and contained for periodic removal by installing a securely anchored trash boom fabricated from lengths of timber or other suitable floats such as styrofoam-filled, thin-walled steel pipe linked with chains.

5. Spillway control devices such as gates and flashboards that are ill-suited, poorly designed, or uncertain of operation are really nothing more than spillway obstructions. They pose a hazard, especially to dams that cannot withstand overtopping flows.

Where floods are seasonally predictable, the control devices can be kept clear of the waterway during the flood season.

The control devices can be eliminated, and the desired storage level established by raising the control section with a wall or sill and the required spilling capacity supplemented by methods described in (1).

Redundant spilling capacity over inoperative closed devices may already exist or can be provided.

Redundant operating systems can be installed that will be activated should the primary system fail or when operating personnel cannot or do not arrive at the control station. Radial gates can be modified by counterweighting and adding automatic operating control systems actuated by the rising reservoir stage that will open the gate at a compound rate sufficient to pass the estimated maximum flood. That system can be further backed up by installing buoyancy chambers on the face of the gate designed to force the gate open by water pressure alone in direct ratio to the rise in reservoir stage. The outflow capacity will be less for the backup system; but, if it is designed to pass the largest flood of a long period of record, the probability of an inoperable gate during the more critical large, routine floods will be greatly diminished without seriously affecting the capacity for unprecedented infrequent occurrences.

6. A spillway located on fine-grained fill materials without carefully designed and constructed invert cutoffs, water stops, and a filtered drain system can be withdrawn from service by closure with an earth embankment extended and bonded to an impervious foundation. A new spillway can be constructed at another, more secure location. Addition of the necessary seepage and piping control features at the existing spillway can also be considered, but the practicality and security of doing so may be quite uncertain. A spillway located over an embankment dam will settle, particularly during an earthquake. If the spillway components cannot conform to the settlement without significant structural damage or impairment as a watertight channel, it can be similarly decommissioned and replaced.

7. The stability of spillway control sections, gate piers, large retaining walls and channel walls can be increased by methods similar to those discussed under the subsection "Concrete and Masonry Dams," item (3).

A distressed reinforced concrete conduit discharge carrier can be strengthened with internal steel sets and a concentric concrete lining if the reduced discharge capacity is acceptable.

Damaged or overstressed radial gate anchorages can be replaced with new post-tensioned trunnion block systems.

8. Gates, valves, hoists, bulkheads, stoplogs, etc., can be removed and disassembled and then refurbished by sand blasting, welding, machining, and otherwise repairing each item. Replacement parts are available or can be custom manufactured. Gate seats and seals can be replaced. New improved gate lifts, hoists, engines, motors, etc., can be obtained. Standby emergency generators can be installed to back up the supply of commercial energy.

9. Concrete deterioration and remedial methods for spillway components are similar to those discussed in the subsection "Concrete and Masonry Dams," item (1).

Outlet Works. As with spillways, there are many types and locations of impoundment outlets. There are tunnels or conduits. There are openings and ports through concrete dams. Outlets disrupt the continuity of the dam or of the foundation. They are internally subjected to reservoir water pressure and can transmit that pressure to the dam or foundation anywhere along their alignment. They are a major source of potential weakness in the dam or foundation, especially in the case of an embankment dam. Some of the more common defects encountered are:

1. Inadequate capacity to lower or control the reservoir stage.
2. Unsafe location of control structure; dangerous or restrictive gating facilities.

3. Unsafe location of outlet conduit.
4. Inadequate control of peripheral seepage.
5. Structural weaknesses.
6. Damaging hydraulic performance characteristics, cavitation, lack of energy-dissipating terminal structures, or unsafe release points.
7. Obstructions to flow.
8. Poorly maintained or inoperative mechanical/electrical features.
9. Deterioration of concrete and metal.

Because of location and surrounding physical constraints, it may be impossible to rehabilitate or modify an outlet. In such a situation, the only practical solution is to construct another one. The existing outlet can be safely removed from service in several ways, depending on the nature and endangerment of the defect and its relationship to the adjacent dam or foundation.

1. An outlet of inadequate capacity can be supplemented with a new one. A new outlet can be constructed on the foundation of an embankment dam by breaching the dam, installing or casting the conduit in place, and replacing the embankment. Proven design and contruction features similar to those for a new, modern project are employed. A tunnel outlet can be driven through an abutment. An opening can be broken through a concrete dam by drilling and pneumatic jacking; a steel conduit or liner installed; the annular space filled with concrete, mortar, or grout; and control facilities installed. The altered stress pattern about larger openings is investigated and reinforcing members added when needed.

If the only defect is inadequate capacity, the old outlet can remain in service. If the outlet is structurally defective, it can be reinforced and kept in service, or it can be plugged with concrete or mortar and grouted to remove it from service. The entire conduit can be filled or the plug can be of limited length and the conduit filled with drain material downstream. If the conduit is removed from service, it may or may not require replacement depending on the need for water service.

2. Outlets beneath embankment dams that are gated only at the downstream end are particularly hazardous because the surrounding embankment and foundation are subjected to full reservoir pressure when the gate is closed. Any leakage from the conduit can result in piping.

Upstream and downstream bifurcations and associated gates and valves can be added to an outlet conduit for safer, more dependable, more flexible control of outflow, and to facilitate otherwise neglected maintenance and repair. Guard gates can be added in line ahead of service or regulating gates.

3. An outlet conduit positioned in the fill of an embankment dam or on a yielding foundation is potentially unsafe, unless it is securely designed for flexibility, axial stretching, and watertightness, and unless the materials will not deteriorate. This potential is especially great for a conduit that crosses a deep embankment foundation cutoff. These outlets can be replaced and safely deactivated as described in (1).

4. Seepage appearing around the exterior of an outlet conduit must be intensively investigated for its source and travel path in order to determine the correct remedial measures.

The conduit can be exposed over a portion of its length near the downstream end and enveloped with drain and filter zones.

The interface and surrounding backfill can be chemically grouted through the walls of larger conduits.

A shaft can be sunk from the surface above the conduit alignment and cutoffs placed about the conduit exterior.

5. A distressed reinforced concrete conduit of larger size can be strengthened as discussed under "Spillways," item (7).

A bare steel conduit of doubtful strength or which may be badly corroded can be strengthened and rehabilitated by centering a smaller pipe or liner inside the conduit and pressure-filling the annular space with mortar. The alignment and grade of the conduit must be reasonably straight and the reduced discharge capacity must be acceptable. Construction details for proven techniques are available.

A dry-type intake tower of doubtful stability or of resistance to flotation can be converted to a more stable wet-type tower by modifying the piping and gating system.

Structural defects in other external outlet works components, such as open channels, intake structures, walls, and energy dissipators can be rehabilitated as described in the subsection "Concrete and Masonry Dams," item (3).

6. Cavitation of conduit surfaces in high velocity outlet works at flow-disrupting locations and at gates and valves can be repaired with resistant materials such as stainless steel liners or epoxy concrete. The fluidway boundary surfaces can be straightened and irregularities removed or smoothed. Air can be introduced where sub-atmospheric pressures are created in the water, especially at gates and valves. Spring points can be formed in the conduit walls for flow separation.

An energy-dissipating terminal structure can be added to control erosion at the outlet release point.

A conduit can be extended to a point of safe release.

A defectively designed or constructed stilling basin can be modified as discussed under "Spillways," item (3).

7. A silted intake structure can be vertically extended by constructing a riser on top of the existing intake.

An actual or incipient slide imperiling an entrance or return channel can be removed or stabilized.

8/9. Defects and rehabilitation measures for mechanical/electrical features and materials deterioration are similar in principle to those discussed under "Spillways," item (8).

Other Considerations

Defects that may be associated with the reservoir basin are:

1. Thin, weak, natural topographic and geologic barriers impounding the reservoir.
2. Large-volume incipient or potential slide masses that can move suddenly at high velocities into the reservoir pool and create water surges that overtop the dam.
3. Economic loss of stored water through pervious geologic structure.

A weak natural barrier of limited topographic expression and extent can be strengthened by seepage control, drainage, and stabilizing measures similar to those employed for embankment dams.

A reservoir that leaks over a large area probably cannot be sealed economically. If the leaking areas are of limited extent and can be selectively identified, it may be possible to reduce the water losses from the reservoir by blanketing those areas with compacted impervious soils, covered by a protective blanket of sand and gravel or fine rock. Reservoir leakage of this nature would not be expected to cause any loss of basin integrity or catastrophic release of storage, except where it might occur in the immediate proximity of the dam or thin natural barriers.

The stability of reservoir slides can be improved by unloading the upper portion of the slide, buttressing the base, drainage, and chemical treatment. The potential for such an event should be examined during the integrity investigation. The freeboard on the dam can be increased some judgmental amount to provide for slide volume and wave generation. The unusual topographic, geologic, and ground water conditions contributing to those very few cases where devastating slides have actually occurred would appear to be extremely rare.

Defects in the following project features, although not directly affecting impoundment integrity, can impede project operation and maintenance, especially during an emergency situation:

1. Impassable or inadequate access roads and bridges.
2. Lack of communication facilities.
3. Lack of emergency lighting at critical locations along spillways, outlets, and the dam crest.

Appropriate rehabilitation methods are obvious.